

**GUIDELINES
FOR
STRUCTURAL EVALUATION
AND
STRENGTHENING OF FLEXIBLE
ROAD PAVEMENTS USING
FALLING WEIGHT DEFLECTOMETER (FWD)
TECHNIQUE**



**INDIAN ROADS CONGRESS
2014**

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GUIDELINES FOR STRUCTURAL EVALUATION AND STRENGTHENING OF FLEXIBLE ROAD PAVEMENTS USING FALLING WEIGHT DEFLECTOMETER (FWD) TECHNIQUE

1 INTRODUCTION

The initial draft on “Guidelines for Structural Evaluation and Strengthening of Flexible Road Pavements using Falling Weight Deflectometer (FWD) Technique” prepared by Prof. K. Sudhakar Reddy, Member, Flexible Pavement Committee (H-2). The Committee deliberated on the draft in a series of meetings. The H-2 Committee finally, approved the draft document in its meeting held on 16th March, 2013. The Highways Specifications & Standards Committee (HSS) approved the draft document in its meeting held on 19th July, 2013. The Executive Committee in its meeting held on 31st July, 2013 approved the same document for placing it before the Council. The Council in its meeting held at New Delhi on 11th and 12th August, 2013 approved the draft “Guidelines for Structural Evaluation and Strengthening of Flexible Road Pavements using Falling Weight Deflectometer (FWD) Technique” for publishing.

The back-calculation software “KGPBACK” was developed by IIT, Kharagpur for back calculation of elastic moduli of the existing pavement layers and the same is available in the CD with these guidelines.

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1.1 Structural evaluation of pavements involves application of a standard load to the pavement and measuring its response in terms of stress, strain or deflection. Benkelman beam has been among the earliest equipment used for measuring deflection and structural evaluation of pavements. In this technique, a static load is applied to the pavement surface and rebound deflections are measured at one or more locations. Measurement of deflection under a static load does not simulate the loading conditions produced in pavements by a moving vehicle. The evaluation of pavements by such methods is labour-intensive and, in general, time consuming. The existing guidelines for strengthening of flexible pavements using Benkelman beam technique are contained in IRC:81 and were developed based on the findings of the Research Scheme R-6 of Ministry of Road Transport and Highways carried out during 1984 to 1990.

1.2 One of the serious limitations of the Benkelman beam technique is that the measurement of deflection under a static load does not simulate the loading conditions produced in pavements by a moving vehicle. Besides, the pavement sections considered under R-6, which forms the basis of the existing guidelines, comprised thin and less stiff bituminous courses like BM and Premix Carpet as opposed to modern pavements comprising thick and stiffer bituminous courses like DBM and BC. Benkelman beam evaluation under static load is known to under-estimate the strength of DBM and BC layers. Thirdly, the Benkelman beam method is labour-intensive and time consuming and test results are likely to be affected by moving traffic on adjacent lanes, and unsafe to be carried out under heavy traffic conditions. Fourthly, the repeatability of test results is always a concern. Finally, this method does not predict the properties of the different constituent layers of the pavement, which is necessary for a comprehensive evaluation of the pavement. On account of these limitations in the existing pavement evaluation methods, it is considered necessary to have a separate set of guidelines, especially for high performing well designed pavements.

1.3 Significant developments have taken place during the past few decades in terms of the equipment and the analytical tools adopted for structural evaluation of pavements. Impulse loading equipment commonly known as Falling Weight Deflectometer (FWD) have been developed, which closely simulates the duration and amplitude of the load pulses

produced by moving wheel loads. Analytical tools to backcalculate the elastic properties of the existing pavement have been developed over the years. One such backcalculation software has been developed by the Transportation Engineering Division of IIT Kharagpur (Reddy, 2003) under the Ministry's Research Scheme R-81. Using the elastic properties of the existing pavement layers, the requirement of strengthening can be worked out following a mechanistic-empirical design procedure. One such procedure and the software have been provided in IRC:37-2012.

2 SCOPE

2.1 These guidelines are meant for evaluating the structural condition of in-service flexible road pavements using deflection data from Falling Weight Deflectometer as well as other pavement data as inputs to a backcalculation model for determining the elastic moduli of pavement layers, and, thereafter, using these moduli as inputs to a pavement design model for estimating the overlay requirement. The backcalculation software used in these guidelines is the one developed as a result of the research carried out under MoRTH Research study (R-81, 2003) "Structural Evaluation of Pavements using Falling Weight Deflectometer" and other studies conducted by the Transportation Engineering Section of IIT Kharagpur (R-81, 2003; Kumar, 2001; Reddy, 2003; Uday Kumar, 2011). The pavement design model used in these guidelines is the one prescribed in IRC:37-2012.

2.2 These guidelines need to be applied to estimate (a) The residual life of an existing pavement and (b) the overlay requirement.

2.3 Structural evaluation exercise should include pavement condition assessment, determining in-situ subgrade strength and pavement material investigations besides deflection testing. The assessment of functional and structural condition of the pavement has to be made on the basis of judicial evaluation of all relevant information collected from the aforementioned investigations.

2.4 These guidelines may require revision from time-to-time based on future experience and developments in the field. Towards this end, it is suggested to all the organizations using the guidelines to keep a detailed record of periodical measurements (both before and after strengthening), type and thickness of overlay provided, performance, traffic, climatic condition, etc. and provide feedback to the Indian Roads Congress for further revision.

3 PRINCIPLE OF PAVEMENT EVALUATION USING FWD

3.1 Performance of flexible pavements can be evaluated by applying loads on the pavements that simulate the traffic loading, recording the response to such loading by measuring the elastic deflection under such loads, and analyzing these data duly considering the factors influencing the performance such as subgrade strength, thickness and quality of each of the pavement layers, drainage conditions, pavement surface temperature etc.

3.2 Among the equipment available for structural evaluation of pavements, the Falling Weight Deflectometer (FWD) is extensively used world-wide because it simulates, to a large

extent, the actual loading conditions of the pavement. When a moving wheel load passes over the pavement it produces load pulses. Normal stresses (vertical as well as horizontal) at a location in the pavement will increase in magnitude from zero to a peak value as the moving wheel load approaches the location. The time taken for the stress pulse to vary from zero to peak value is termed as 'rise time of the pulse'. As the wheel moves away from the location, magnitude of stress reduces from peak value to zero. The time period during which the magnitude of stress pulse varies from 'zero-to-peak-to-zero' is the pulse duration. Peak load and the corresponding pavement responses are of interest for pavement evaluation.

3.3 The resulting load-deflection data can be interpreted through appropriate analytical techniques, such as backcalculation technique, to estimate the elastic moduli of the pavement layers. The computed moduli are, in turn, used for (i) the strength evaluation of different layers of in-service pavements (ii) the estimation of the remaining life of in-service pavement (iii) determination of strengthening requirement, if any and (iv) evaluation of different rehabilitation alternatives (overlay, recycling, partial reconstruction, etc).

4 FALLING WEIGHT DEFLECTOMETER

4.1 Falling Weight Deflectometer (FWD) is an impulse-loading device in which a transient load is applied to the pavement and the deflected shape of the pavement surface is measured. The working principle of a typical FWD is illustrated in **Fig. 1**. D_0 , D_1 , etc., mentioned in **Fig. 1** are surface deflections measured at different radial distances. Impulse load is applied by means of a falling mass, which is allowed to drop vertically on a system of springs placed over a circular loading plate. The deflected shape of the pavement surface is measured using displacement sensors which are placed at different radial distances starting with the center of the load plate. Trailer mounted as well as vehicle mounted FWD models are available commercially. The working principle of all these FWD models is essentially the same. A mass of weights is dropped from a pre-determined height onto a series of springs/buffers placed on top of a loading plate. The corresponding peak load and peak vertical surface deflections at different radial locations are measured and recorded.

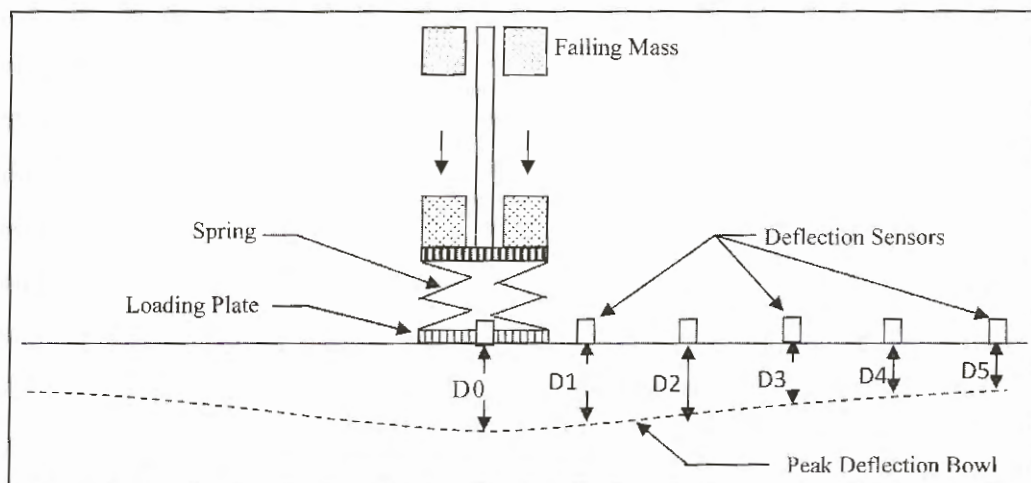


Fig. 1 Working Principle of Falling Weight Deflectometer

4.2 Different magnitudes of impulse load can be obtained by selection of a suitable mass and an appropriate height of fall. Under the application of the impulse load, the pavement deflects. Velocity transducers are placed on the pavement surface at different radial locations to measure surface deflections. Geophones or seismometers are used as displacement transducers. Load and deflection data are acquired with the help of a data acquisition system.

4.3 Typical Falling Weight Deflectometers (FWD) include a circular loading plate of 300 or 450 mm diameter. In these guidelines 300 mm diameter load plate is recommended. A rubber pad of 5 mm minimum thickness should be glued to the bottom of the loading plate for uniform distribution of load. Alternatively, segmented loading plates (with two to four segments) can be used for better load distribution.

4.4 A falling mass in the range of 50 to 350 kg is dropped from a height of fall in the range of 100 to 600 mm to produce load pulses of desired peak load and duration. Heavier models use falling mass in the range of 200 to 700 kg. The target peak load to be applied on bituminous pavements is 40 kN (+/- 4 kN), which corresponds to the load on one dual wheel set of a 80 kN standard axle load. The target peak load can be decreased suitably if the peak maximum (central) deflection measured with 40 kN load exceeds the measuring capacity of the deflection transducer. Similarly, the load can be increased to produce deflection of at least 10 μ m at a radial distance of 1.2 m. If it is known from construction records or from coring or from test pits that subgrade is stiff and hence smaller than 10 μ m deflections are expected, testing with increased loads will not be required. If the applied peak load differs from 40 kN, the measured deflections have to be normalized to correspond to the standard target load of 40 kN. The normalization of deflections can be done linearly. For example, if the measured deflection is 0.80 mm for an applied peak load of 45 kN, the normalized deflection for a standard load of 40 kN is 0.711 mm ($0.80 * (40/45)$). The load cells used to measure load pulses produced by FWD should have a reading resolution of 0.1 kN or better and should give readings accurate to 2 percent of measured value.

4.5 The stiffness of bituminous layers and hence the response of a pavement depends on the pulse shape of the applied load (COST 336, 2005). Most FWDs have a load rise time (from start of pulse to peak) of between 5 ms and 30 ms and have a load pulse base width in the interval of 20 ms to 60 ms (COST 336, 2005). The duration of impulse load is maintained approximately equal to the time needed to traverse the length of a tyre imprint at a speed of about 60 km/h which is in the range of 20 to 30 ms. The FWDs used for evaluation should be capable of producing load pulses with loading time in the range of 15 to 50 ms.

4.6 Sufficient number of deflection transducers should be used to adequately capture the shape of deflection bowl. Six to nine velocity transducers (geophones) are generally adequate for measuring surface deflections of flexible pavements. Deflection sensors are placed on the surface of pavement at different radial direction aligned in the longitudinal direction. The deflection transducers used should have a reading resolution of at least 1 μ m and

should be accurate to ± 2 percent of the reading. Typical geophone position configurations (number and radial distances measured from center of load plate) commonly used for flexible pavement evaluation are :- (i) 7 sensors at 0, 300, 600, 900, 1200, 1500 and 1800 mm radial distances (ii) 7 sensors at 0, 200, 300, 450, 600, 900, 1500 mm radial distances (iii) 6 sensors at 0, 300, 600, 900, 1200 and 1500 mm radial distances and (iv) 6 sensors at 0, 200, 300, 600, 900, 1200 mm radial distances.

4.7 Calibration of the FWD

It is essential that FWDs are calibrated for getting accurate and reproducible results.

4.7.1 *Static calibration*

The load cell(s) used in the FWD should be calibrated in a standard laboratory and the readings of the load cell(s) should be matched to those of the reference load cell. The readings of the FWD load cell(s) should be accurate to 2 percent of the reference load cell readings. The date of calibration of the load cell should not be earlier than 365 days from the date of structural evaluation of pavements using FWD.

4.7.2 *Load repeatability*

For this test, FWD measurements should be carried out on a level bituminous pavement surface, which does not have any cracking. The range of load applied should generate peak central deflections in the range of 250 μm to 600 μm . The standard deviation of the peak load in the load repeatability test estimated from a minimum of twelve load drops should be less than 5 percent of the mean value of peak load. Load repeatability may be checked before using the FWD for any major investigation.

4.7.3 *Absolute calibration of deflection transducers*

Dismounted deflection transducers should be calibrated in a laboratory setup following any approved procedure and the deflection transducers should be accurate to 2 percent of reference deflections. The date of static calibration of geophones should not be earlier than one year from the date of structural evaluation of pavements using the FWD.

Deflection repeatability check may also be conducted using the data collected in load repeatability test. The standard deviation of the normalized deflections, should be less than 5 percent of the mean value of the reading.

Relative deflection comparison may be done before using the FWD for evaluation in a project. This can be done by stacking all the transducers, one above the other, in a suitable stand and placing the stand on the pavement surface (level and free from cracks). Deflection readings of all the transducers corresponding to a series of load drops are recorded and compared. The deflections produced in this test should be in the range of 250 μm to 600 μm . Difference between maximum and minimum of the recorded (normalized) deflections should be within 4 μm .

5 PAVEMENT EVALUATION SURVEY AND DATA COLLECTION

5.1 General

The following are the broad categories of survey, investigation and data collection exercise; (i) historical data about the pavement (ii) condition survey of the pavement for identification of uniform sections having similar conditions, (iii) deflection measurements using falling weight deflectometer and (iv) pavement layer thickness and composition and the subgrade characteristics.

5.2 Historical Data

Historical data on pavement can be useful in identifying the reasons for different distresses and in establishing whether the distresses were caused by deficiency in design, poor material selection, improper construction and other reasons such as high water table and poor drainage. Even though in most cases it would be difficult to have a comprehensive compilation of pavement history, it would be advisable to look for the records to find out the following:

- i) Initial pavement design – basis of design, guidelines followed, performance criteria adopted, layer types, layer thicknesses and properties assigned
- ii) Data regarding functional and structural evaluation carried out if the existing surface is an overlay
- iii) Laboratory testing results on different materials used
- iv) Internal and external drainage arrangement made
- v) 'As constructed' pavement details
- vi) Construction methods adopted
- vii) Quality control test results
- viii) Maintenance and rehabilitation data such as past distresses, effectiveness of different maintenance and rehabilitation measures undertaken in the past
- ix) Climatic condition data regarding rainfall intensity, temperature, occurrence of floods, overtopping of pavement, high water table, etc.
- x) Past traffic volume history, axle load spectrum data
- xi) Discussions with the engineers associated with design, construction and maintenance stages can yield valuable information.

5.3 Pavement Condition Survey

5.3.1 Pavement condition survey shall precede the actual deflection measurement and consists primarily of visual observations supplemented by measurements for estimation of cracking, rutting and other distresses in the pavement. It may be prudent to identify possible

causes of distress using condition survey and other investigations such as coring, test pits and laboratory testing before conducting extensive deflection measurements using falling weight deflectometer. If the distresses are not related to the structural capacity of pavement layers, FWD measurements may not be of much help. Rutting in bituminous mix, distress caused by stripping in mix, etc. are situations in which the structural condition of the pavement may not necessarily be explained adequately by FWD measurements.

5.3.2 The information collected in the condition survey should be relevant and adequate for identifying sections of uniform performance for the purpose of determining the sample size for deflection measurements. Homogeneous sections for overlay design purpose will be identified on the basis of deflection measurements and other relevant inputs such as traffic, subgrade characteristics, deflection bowl parameters, etc.

5.3.3 The condition data would help to figure out the causes of distresses, some of which could be treated by non-structural treatments like sealing, slurry surfacing, cold planing and replacement of un-sound materials. This will make up the functional loss and prepare the pavement for receiving the structural overlay. Strengthening of flexible pavements should not be attempted without restoring its functional loss.

5.3.4 Suitable mapping formats should be used for mapping pavement distress data. Each lane of the carriageway and the shoulder should be divided separately into blocks of 50 m length and one-lane width (3.5 m) and distress should be recorded for each block. Paved shoulders should be divided into blocks of 50 m length and pavement shoulder width. The formats should typically capture the following data about the pavement condition besides having information such as pavement chainage, direction of traffic and pavement surface type.

- i) Distress type – cracking, rutting, shoving, bleeding, stripping, raveling, pot holes, patching, etc.
- ii) Severity of distress
- iii) Extent of distress
- iv) Location of distress

Format for pavement condition survey for identifying sections of uniform performance is given as **Appendix-I**.

5.3.5 Based on the data collected from condition survey, the road length shall be classified into sections of uniform performance in accordance with the criteria given in **Table 1**. Rut depth should be measured under a 3 m straight edge in the middle of the sub-section of selected length (50 m). Similarly, cracking and other distresses should be recorded for each block as mentioned in Clause 5.3.4. Sub-sections of uniform performance should be identified for each traffic lane and shoulder separately. This will be useful for

selecting the sampling size for different portions of carriageway for conducting deflection measurements. Identification of sections of “good, fair and poor” performance may be made separately for each lane and shoulder based on the corresponding distress maps.

5.3.6 As it is inexpedient to change the sample size for deflection measurement at frequent intervals, it will be preferable if the length of each uniform section is kept at a minimum of 1 km except in the case of localised failures or in other situations requiring closer examination where minimum length of section should be 0.3 km from the consideration of profile correction and constructability.

Table 1 Criteria for Classification of Pavement Sections

Classification	Pavement Condition
Good	Isolated cracks of less than 3.0 mm width in less than 5% area of total paved surface AND average rut depth less than 10 mm
Fair	Isolated or interconnected cracks of less than 3.0 mm width in 5 to 20% area of total paved surface AND/OR average rut depth between 10 to 20 mm
Poor	Wide interconnected cracking of more than 3.0 mm width in 5 to 20% area (include area of patching and raveling in this) of paved area OR cracking of any type in more than 20% area of paved surface AND/OR average rut depth of more than 20 mm

5.4 Deflection Measurement

5.4.1 Estimation of sample size

The sample size or the interval at which deflection measurements are to be made can be decided using simple statistical principles. Sample size should be larger (i) if the variation in the measured deflections is expected to be large and (ii) if the margin of error between the mean deflection estimated from the measurements and the true mean of deflections should be small. The following equation can be used for estimating the sample size (n).

$$n = (z * CV)^2 / (ME^2) \quad \dots (1)$$

where,

- n = sample size
- z = normalized normal deviate which can be obtained from standard statistical tables for a selected confidence level
- CV = coefficient of variation of deflection (standard deviation/mean) expressed as percentage
- ME = acceptable margin of error (as percentage of mean)

5.4.2 It is recommended that 90 percent confidence level and 10 percent margin of error (ME expressed as % of mean) may be considered in these guidelines for the purpose of estimation of sample size. The coefficient variability values for “good”, “fair” and “poor”

sections may be taken as (AASHTO, 1993) 15 percent, 30 percent and 45 percent respectively. Thus, for 90 percent (two-sided) confidence level (standard normal deviate $z = 1.285$) and 10 percent margin of error, the minimum sample sizes for “good”, “fair” and “poor” sections will be 4, 15, 33. It may be noted that sample size requirement can be different for individual lanes as the condition is likely to be different. However, it is recommended that for a uniform section (good/fair/poor) the sample size required for the most distressed lane be considered for the complete carriageway. The interval at which data should be collected will depend on the length of uniform section. If, for example, the length of section of uniform “fair” performance is 2.0 km, the maximum spacing at which deflections should be measured is $2000/15 = 133$ m. The spacing can be rounded off to convenient practical values.

5.4.3 Different measurement schemes can be adopted. These include (i) measurement along the most distressed wheel path of the carriageway (ii) measurement along inner as well as outer wheel paths of all the lanes (iii) measurement along both wheel paths of only the outer most lanes and (iv) measurement along the more distressed wheel paths of each of the lanes. The guidelines given in **Table 2** are recommended for selection of deflection measurement schemes for different types of carriageways.

Table 2 Guidelines for Selection of Deflection Measurement Scheme

Type of Carriageway	Recommended measurement scheme	Maximum Spacing ¹ (m) for test points along selected wheel path for pavements of different classification		
		Poor	Fair	Good
Single-lane two-way	i) measure along both outer wheel paths	60	130	500
Two-lane two-way single carriageway	i) measure along both outer wheel paths	60	130	500
Four-lane single carriageway	i) measure along outer wheel paths of outer lanes	30	65	250
	ii) measure along the outer wheel path of more distressed inner lane	60	130	500
	iii) measure along the centre line of paved shoulder (in case of widening projects) ²	120	260	500
Four-lane Dual (divided) carriageway (Measurement scheme given for each carriageway)	i) measure along outer wheel paths of outer lanes	30	65	250
	ii) measure along the outer wheel path of inner lane	60	130	500
	iii) measure along the centre line of paved shoulder (in case of widening projects) ²	120	260	500

Dual carriageways with 3 or more lanes in each direction (Measurement scheme given for each carriageway)	i) measure along outer wheel paths of outermost lanes	30	65	250
	ii) measure along outer wheel path of more distressed inner lane	60	130	500
	iii) measure along the centre line of paved shoulder (in case of widening projects) ²	120	260	500

- Notes :**
- 1) The spacings given in the table are with the assumption that the length of uniform section is 1.0 km. The actual spacing to be adopted can be obtained by multiplying the spacing given in the table by the length of uniform section
 - 2) Deflections may be measured along the hard shoulders if the same are proposed to form part of the new lane in case of widening projects

5.4.4 The wheel path selected for deflection study has to be clearly indicated in the data sheets used for recording the deflection data. The scheme of wheel paths and sample size selected for measurement of deflections should not be changed within a section of uniform performance (good, fair, poor).

5.4.5 Positions of wheel paths must be identified by observing the surface condition of the road. If the same cannot be done, the following guidelines can be used for identifying the outermost wheel path.

Outer wheel paths of outer lanes :-

- i) For single-lane two-way carriageway :- at 0.6 m from the outer edge of the outer lane
- ii) For two-lane two-way carriageway and for multi-lane single carriageway :- at 1.0 m from the outer edge of outer lane
- iii) For divided carriageways with two or more lanes in each direction :- 0.75 m from the outer edge of outer lane

Outer wheel paths of inner lanes :-

- i) For multi-lane single carriageway :- at 4.0 m from the outer edge of outer lane
- ii) For divided carriageways with two lanes in each direction 4.2 m from the outer edge of outer lane
- iii) For divided carriageways with three lanes in each direction 4.2 m from the outer edge of outer lane for central lane and at 5.2 m from the outer edge of outer lane for the lane adjacent to median

5.4.6 *The following data should be recorded for each test point.*

- i) section identity (chainage)
- ii) position of lane in the carriageway (outer, inner, etc.)

- iii) transverse position of test point in the lane
- iv) measurement spacing
- v) time of test
- vi) air temperature, pavement surface temperature and/or pavement temperature at a depth of 40 mm
- vii) drop number
- viii) peak values of load and deflections for each drop of load
- ix) whole time history of load and deflections should be stored for one of the test points of each road section
- x) deflection transducer configuration selected (number of sensors and radial offsets from center of load plate)
- xi) loading plate diameter

5.4.7 *Steps for measuring deflection at a test point*

The following steps may be followed for measuring deflections at each test point. The exact sequence of operations may be different for different models of FWD.

- i) Mark the test point on the pavement
- ii) Centre the load plate of the duly calibrated FWD over the test point
- iii) Lower the loading plate onto the pavement. There should be no standing water (surface texture completely filled with water) on the pavement surface. The loading plate should be in proper contact with pavement surface. If a non-segmental plate is used the presence of rutting at test location should be noted if it affects the contact between plate and pavement surface. The longitudinal and transverse slope of the pavement should not exceed 10 percent at the test location for accurate measurement of deflection
- iv) Lower the frame holding the displacement transducers (geophones) so that the transducers are in contact with pavement surface
- v) Raise the mass to a pre-determined height required for producing a target load of 40 kN
- vi) Drop one seating load. Load and deflection data for seating load drop need not be recorded
- vii) Raise the mass and drop. Record load and deflection data into the computer through data acquisition system. While peak load and peak deflections at different selected radial positions must be recorded, complete time history of load and deflections can be stored for each load drop if feasible
- viii) Repeat step vii atleast two more times

- ix) If, during steps vii and viii, the deflections measured are too large or too small as discussed in Clause 4.4, the test may be repeated by changing the peak load
- x) Raise the geophone frame and load plate and move to the next test location
- xi) Record air temperature at half hourly interval
- xii) Record pavement surface temperature (optional) if non-contact temperature sensors are available
- xiii) Measure pavement surface layer temperature at half-hourly intervals by drilling holes of 40 mm depth into the pavement surface layer. Fill the hole with a drop of glycerol. Insert the thermometer into the hole and record the temperature after three minutes
- xiv) Deflection measurements should not be made when the pavement temperature is more than 45°C. Guidelines given in Clause 6.4.3 may be followed for deflection measurement in colder areas and areas of altitude greater than 1000 m

Proforma for recording pavement deflection data is given as **Appendix-II**.

5.5 Determination of Pavement Layer Thicknesses

5.5.1 Pavement layer thicknesses are essential inputs to the process of backcalculation of layer moduli and, in turn, to the estimation of remaining life and overlay requirements of the in-service pavement. Hence, it is necessary that accurate information is collected about layer thicknesses from different sources. Layer thicknesses can be obtained from historical data, by coring bound layers and/or by excavating test pits and/or through the non-destructive technique of Ground Penetrating Radar (GPR) survey.

5.5.2 As it has generally been difficult to get accurate records of as-constructed layer thicknesses, the most effective method of determining thicknesses of all the layers has been the excavation of test pits at suitable spacing and measure the layer thicknesses. Samples of different layer materials can be collected from the test pits which can be examined for signs of degradation and contamination of granular layers, stripping of bituminous mixes, identification of rutted layers. The samples can be tested in the laboratory for evaluating the layer moduli and for exploring causes of distresses, especially in bituminous mixes. This information will be useful for validating the layer moduli backcalculated from analysis besides being useful in explaining the causes for some of the distresses observed on the surface.

5.5.3 It is recommended that 0.6 m x 0.6 m test pits be excavated at 1.0 km interval or at suitable larger interval where other records suggest uniformity of pavement composition in such larger sections. The test pits may be excavated along the outer lanes only starting from the outside edge of the outer lane in the earthen shoulders exposing pavement layers sufficiently to note the condition and thickness of each layer. At each test pit, the number

of layers encountered, description of layer materials including signs of distress, defects, layer thicknesses, interface conditions should be recorded. After collecting necessary data from the test pits they should be filled back as soon as possible with suitable material and compacted so that they do not adversely affect the structural condition of the pavement and also do not create traffic safety hazard. During the period the test pits are excavated and remain unfilled, the area of excavation should be cordoned off by suitable barricading and adequately displayed through traffic safety devices for ensuring night visibility.

5.5.4 Dynamic Cone Penetrometer (DCP) tests may be conducted on the subgrade layer exposed in the test pits to obtain the Dynamic Cone Penetrometer value for in-situ subgrade. The DCP values obtained with a 60° cone can be used to estimate the backcalculated modulus value of subgrade layer using equation 2 (Kumar, 2001). These values can be used for selection of modulus range for subgrade in the backcalculation process.

$$E_{\text{subgrade}} = 357.87 * (\text{DCP})^{-0.6445} \quad \dots (2)$$

where,

E_{subgrade} = backcalculated modulus of subgrade (MPa)

DCP = Dynamic Cone Penetrometer value (mm/blow)

5.5.5 Since it is generally difficult to excavate test pits in the inner lanes, it is suggested that cores be taken in the bituminous layers at 2.0 km interval on the inner lanes and at 1.0 km interval on the outer lanes (in the case of multi-lane divided or undivided carriageways). In addition, cores may also be taken at locations, where structural problems are observed based on evaluation of FWD and/or distress data. These cores may be compared to the cores taken from portions of pavement without any significant distress to find the reasons of deficiencies. The cores should be examined for type of layers and thickness, crack propagation (from bottom or top and depth of crack), and de-lamination in the cores. Stiffness modulus of the bituminous mix can also be determined by conducting appropriate test (such as ASTM D 7369-09) on the core.

5.5.6 The locations of test pits and cores should coincide with the locations of FWD test.

6 ANALYSIS OF DATA

6.1 Processing of Load and Deflection Data

6.1.1 The FWD test data collected from different load drops at each test point primarily consist of peak load, peak deflections at different radial locations. Unrealistic deflection values and obviously erroneous data must be removed. Some of the checks that should be applied to the deflection data are :- (i) deflections should decrease with increasing distance from the loading plate and (ii) deflection values should not be more than the capacity of the sensors. Average values of load and deflections are calculated from the three drop test data collected at a given location.

6.1.2 The deflections are normalized to correspond to a standard target load of 40 kN as explained in Clause 4.4.

6.2 Identification of Homogeneous Sub-sections

6.2.1 The identification of sections of uniform performance done in Section 4 of these guidelines was done primarily to select an appropriate sample size for conducting deflection testing. Since the assessment of the remaining life of existing pavement and the strengthening requirement in terms of bituminous overlay will be done on the basis of the backcalculated moduli of in-service pavement layers, it is prudent to identify homogeneous sections for the purpose of structural design primarily based on deflection bowl parameters and other relevant information.

6.2.2 Identification of homogeneous sections is generally done on the basis of the following parameters :- peak deflections or peak deflection bowl parameters, subgrade strength, design traffic, layer thicknesses and extent and severity of distress, backcalculated surface modulus of the total bituminous layers, remaining life of pavement and overlay thickness requirement. It is proposed in these guidelines that one of the deflection bowl parameters, which typically represent the stiffness of the upper layers along with design traffic and subgrade strength, should be used for identification of homogeneous sections. Other parameters as may be deemed suitable can also be considered for this purpose. Surface Curvature Index (SCI) calculated as the difference between D_0 and D_{300} where, D_0 and D_{300} are the peak deflections (mm) measured at the center of loading plate and at a radial distance of 300 mm is a bowl shape parameter, which reflects the contribution of upper layers, is the bowl shape parameter to be used, alongwith other parameters, for identification of homogeneous sections. SCI is expressed in mm here whereas the parameter is used in inches or mils in many empirical expressions available in literature for empirically estimating moduli of layers.

6.2.3 A statistical technique popularly used for identification of homogeneous sections is the "Cumulative Difference" approach. This approach is already being used extensively in India in many highway projects. In this approach, the sequence of actual cumulative sums in a measurement series is compared with the sums that would have resulted from adding averages. The difference between these values is termed as cumulative difference. The series of cumulative differences (z_k) for the measured sequence of a given variable 'x' (SCI, subgrade strength, etc.) can be obtained using the following expression.

$$z_k = \sum_{i=1}^{i=k} (xi - k\bar{x}) \quad \dots (3)$$

for all $k = 1, \dots, n$,

where $\bar{x} = (1/n) \sum_{i=1}^{i=n} (xi)$

Wherever the trend changes from positive to negative and vice-versa in the plot of cumulative difference VS distance (or number of test location), that should be considered as a possible

delineator for identifying homogeneous sections. However, judgment has to be applied for considering a particular change in trend to be significant enough to suggest the presence of a delineator there.

6.2.4 Homogeneous sections can be identified with reference to different parameters such as SCI, traffic, subgrade strength, etc. Delineation carried out based on different parameters will yield a number of sub-sections. No sub-section should be shorter than 1.0 km in length and each subsection should have atleast twelve deflection test locations. If a subsection has only one or two test points, it is a case of the pavement in need of localized rehabilitation measures. The spacing considered for deflection measurement in each sub-section can be rounded off to convenient practical values.

6.3 Backcalculation of Layer Moduli

6.3.1 Measured surface deflections, normalized to a standard load of 40 kN, alongwith other inputs such as radial distances at which deflections are measured, layer thicknesses, Poisson's ratio values of different layers, applied peak load and loading plate radius, are used to backcalculate the elastic moduli of different layers of the existing pavement using an appropriate backcalculation technique. The backcalculated moduli are used for the analysis of the in-service pavement and for assessment of the structural condition of the pavement. KGPBACK, which is a specific version of BACKGA program developed by the Transportation Engineering Section of IIT, Kharagpur for the research scheme R-81 (2003) of the Ministry of Road Transport and Highways, is recommended in these guidelines for backcalculation. Details of KGPBACK are given in **Appendix-III**.

6.4 Correction for Temperature

6.4.1 Backcalculated moduli values of the bituminous layers evaluated by FWD survey are influenced by the pavement temperature. Hence the backcalculated moduli obtained at temperatures other than the identified standard temperature will have to be corrected. For areas in India having a tropical climate, the standard pavement temperature is recommended as 35°C.

6.4.2 The backcalculated modulus of bituminous layer obtained from deflection survey conducted at a temperature " T_2 " °C can be corrected to estimate the modulus corresponding to a temperature of " T_1 " °C using equation 4 (Reddy, 2003).

$$E_{T_1} = \lambda E_{T_2} \quad \dots (4)$$

where,

λ , temperature correction factor, is given as

$$\lambda = \frac{1 - 0.238 \ln T_1}{1 - 0.238 \ln T_2} \quad \dots (5)$$

where,

E_{T_1} = backcalculated modulus (MPa) at temperature T_1 (°C)

E_{T_2} = backcalculated modulus (MPa) at temperature T_2 (°C)

The above factor was developed for a pavement temperature range of 25 to 40°C. The trends obtained by Reddy (2003) and Uday Kumar (2011) suggest that the relationship can be extrapolated upto a temperature range of 20 to 45°C. Temperature correction need not be applied to backcalculated modulus values of thin bituminous layers (less than 40 mm) and for “Poor” sections.

6.4.3 In colder areas and areas of altitude greater than 1000 m where the average daily temperature is less than 20°C for more than 4 months in a year, the standard pavement temperature of 35°C will not apply. In the absence of adequate data about deflection-performance relationship, it is recommended that deflection measurements in such areas be made when the ambient temperature is greater than 20°C. No temperature correction needs to be applied for backcalculated moduli of bituminous layers in this case.

6.5 Correction for Seasonal Variation

6.5.1 Moisture content affects the strength of subgrade and granular subbase/base layers. The extent to which the strength is affected will depend on the nature of subgrade soil, gradation and nature of fines in the granular layers, etc. For the purpose of applying these guidelines, it is intended that the pavement layer moduli values should pertain to the period when the subgrade is at its weakest condition. In India, this period occurs during the recession of monsoon. It is, therefore, desirable to conduct deflection measurements during this period. Where the same is not feasible, a correction procedure should be adopted.

6.5.2 Relationships developed (R-81, 2003; Reddy, 2003) based on FWD studies conducted on different highways in eastern India are adopted in these guidelines. Until data regarding the seasonal variation of backcalculated granular and subgrade moduli are available from other zones of the country, it is recommended that the equations developed in R-81 research scheme be adopted. Equations 6 and 7 can be used for estimating the modulus value of subgrade layer from the modulus value backcalculated from deflections collected during winter and summer respectively. Equations 6 and 7 are applicable for monsoon subgrade modulus of 20 MPa or more (Winter/Summer Modulus of 30 MPa). Similarly, 8 and 9 are applicable for monsoon granular layer modulus of 60 MPa or more (Minimum Winter modulus of 80 MPa and minimum Summer Modulus of 100 MPa).

$$E_{\text{sub_mon}} = 3.351 * (E_{\text{sub_win}})^{0.7688} - 28.9 \quad \dots (6)$$

$$E_{\text{sub_mon}} = 0.8554 * (E_{\text{sub_sum}}) - 8.461 \quad \dots (7)$$

where,

$E_{\text{sub_mon}}$ = subgrade modulus in monsoon (MPa)

$E_{\text{sub_win}}$ = subgrade modulus in winter (MPa)

$E_{\text{sub_sum}}$ = Subgrade modulus in summer (MPa)

Seasonal correction factors for granular layers are given by equations 8 and 9.

$$E_{\text{gran_Mon}} = -0.0003 * (E_{\text{gran_Sum}})^2 + 0.9584 * (E_{\text{gran_Sum}}) - 32.989 \quad \dots (8)$$

$$E_{\text{gran_Mon}} = 10.5523 * (E_{\text{gran_win}})^{0.624} - 113.857 \quad \dots (9)$$

where,

$E_{\text{gran_mon}}$ = granular layer modulus in monsoon (MPa)

$E_{\text{gran_win}}$ = granular layer modulus in winter (MPa)

$E_{\text{gran_sum}}$ = granular layer modulus in summer (MPa)

7 ESTIMATION OF DESIGN TRAFFIC

7.1 General

7.1.1 Traffic, in terms of standard axle load (80 kN) repetitions, shall be considered for design of overlay. The cumulative standard axle repetitions may be worked out based on actual data. Otherwise, the design traffic may be calculated as per the procedure given in IRC:37 and Clause 7.4 below. For the purpose of structural design of pavements, only the number of commercial vehicles of laden weight of 3 tonnes or more will be considered. The traffic is considered in both directions in the case of two lane road and in the direction of heavier traffic in the case of multi lane divided highways. To obtain a realistic estimate of design traffic due consideration should be given to the existing traffic, possible changes in road network and land-use of the area served, the probable growth of traffic and design life. Estimate of the initial daily average traffic flow for any road should normally be based on 7-day 24-hours classified traffic counts. However, in exceptional cases where this information is not available 3-day count may be used.

7.2 Traffic Growth Rate

An estimate of likely growth rate can be obtained as follows:

- a) by studying past trends in traffic growth.
- b) by establishing econometric models, as per the procedure outlined in IRC guidelines

If the data for the annual growth rate of commercial vehicles is not available or if it is less than 5 percent, a growth rate of 5 percent shall be used (IRC:SP:84).

7.3 Design Life

The design life is defined in terms of the cumulative number of standard axles in msa that can be carried before a major strengthening, rehabilitation or capacity augmentation of the pavement is necessary. It is recommended that design life of overlay should be

10 years. However, less important roads may be designed for a shorter design period but not less than 5 years in any case.

7.4 Computation of Design Traffic

7.4.1 The design traffic is considered in terms of the cumulative number of standard axles (in the lane carrying maximum traffic) to be carried during the design life of the road. This can be computed using the following equation:

$$N = \frac{365 \times [(1+r)^n - 1]}{r} \times A \times D \times F \quad \dots (10)$$

where,

- N = cumulative number of standard axles to be catered for in the design in terms of million standard axles, msa
- A = Initial traffic in the year of completion of construction, in terms of number of Commercial Vehicles Per Day (CVPD)
- D = Lane distribution factor (as explained in Clause 7.4.2)
- F = Vehicle Damage Factor (VDF)
- n = Design life in years
- r = Annual growth rate of commercial vehicles expressed in decimal (eg : for 5 percent annual growth rate, $r = 0.05$)

The traffic in the year of completion is estimated using the following formula.

$$A = P (1+r)^x \quad \dots (11)$$

where,

- P = Number of commercial vehicles as per last count
- x = Number of years between the last count and the year of completion of construction

7.4.2 Distribution of commercial traffic over the carriageway

A realistic assessment of the distribution of commercial traffic by direction and by lane is necessary as it directly affects the total equivalent standard axle load applications used in the design process. In the absence of adequate and conclusive data, it is recommended that the following distribution may be assumed for design until more reliable data on placement of commercial vehicles on the carriageway lanes are available. However, if in a particular situation a better estimate of the distribution of traffic between the carriageway lanes is available from traffic surveys, the same should be adopted and the design should be based on the traffic in the most heavily trafficked lane. The design will normally be applied over the whole carriageway width.

i) Single-lane roads (3.75 m width)

Traffic tends to be more channelized on single-lane roads than two-lane

roads and to allow for this concentration of wheel load repetitions, the design should be based on total number of commercial vehicles in both directions.

ii) **Two-lane single carriageway roads**

The design should be based on 50 percent of the total number of commercial vehicles in both directions. If vehicle damage factor in one direction is higher, the design traffic in the direction of higher VDF is recommended.

iii) **Four-lane single carriageway roads**

The design should be based on 40 percent of the total number of commercial vehicles in both directions.

iv) **Dual carriageway roads**

The design of dual two-lane carriageway roads should be based on 75 percent of the number of standard axles in each direction. For dual three-lane carriageway and dual four-lane carriageway, the distribution factor will be 60 percent and 45 percent respectively.

Where there is no significant difference between the traffic in the two directions, the design traffic for each direction may be assumed to be as half of the sum of traffic in both directions. Where significant difference between the two streams exists, pavement thickness in each direction can be different and designed accordingly.

For two way two lane roads, pavement thickness should be same for both the lanes even if VDF values are different in different directions and designed for higher VDF. For divided carriageways, each direction may have different thickness of pavement if the axle load patterns are significantly different.

Where the distribution of traffic between the carriageway lanes and axle load spectrum for the carriageway lanes are available, the design should be based on the traffic in the most heavily trafficked lane and the same design will normally be applied for the whole carriageway width.

7.4.3 *Vehicle damage factor*

7.4.3.1 The Vehicle Damage Factor (VDF) is a multiplier for converting the number of commercial vehicles of different axle loads to equivalent number of standard axle load (80 kN) repetitions. It gives the equivalent number of standard axles per commercial vehicle. The vehicle damage factor is arrived at from axle load surveys conducted on typical road sections so as to cover various influencing factors such as traffic mix, type of transportation, type of commodities carried, time of the year, terrain, road condition and degree of enforcement. The AASHO axle load equivalence factors may be used for converting the axle load spectrum to an equivalent number of standard axles. For designing a strengthening layer on an existing road pavement, the vehicle damage factor should be arrived at carefully by using the relevant available data or carrying out specific axle load surveys depending upon importance of the project. Minimum sample size to be considered for collecting axle load data is given in **Table 3**. Axle load survey should be carried out without any bias for loaded

or unloaded vehicles. On some sections, there may be significant difference in axle loading in the two directions of traffic. In such situations, the VDF should be evaluated direction-wise for the purpose of design. Each direction can have different overlay thickness for divided carriageways depending upon the loading pattern. The Spectrum of axle loads should be determined separately for single, tandem, tridem and multi axle loads.

Table 3 Sample Size for Axle Load Survey

Total Number of Commercial Vehicles per day	Minimum Percentage of Commercial Traffic to be Surveyed
<3000	20%
3000 to 6000	15%
>6000	10%

7.4.3.2 Equations 12 to 15 are used to compute equivalent axle load factors for single axle with a single wheel on both sides, single axle with dual wheel sets on both sides, tandem axle and tridem axle respectively.

$$\text{Single axle with single wheel on both sides} = \left(\frac{\text{axle load in kN}}{65} \right)^4 \quad \dots (12)$$

$$\text{Single axle with dual wheels on both sides} = \left(\frac{\text{axle load in kN}}{80} \right)^4 \quad \dots (13)$$

$$\text{Tandem axle with dual wheels on both sides} = \left(\frac{\text{axle load in kN}}{148} \right)^4 \quad \dots (14)$$

$$\text{Tridem axles with dual wheels on both sides} = \left(\frac{\text{axle load in kN}}{224} \right)^4 \quad \dots (15)$$

7.4.3.3 Where sufficient information on axle loads is not available and the project size does not warrant conducting an axle load survey, the indicative values of vehicle damage factor as given in **Table 4** may be used.

Table 4 Indicative VDF Values

Initial Traffic Volume in Terms of Commercial Vehicles Per Day	Terrain	
	Rolling/Plain	Hilly
0-150	1.5	0.5
150-1500	3.5	1.5
More than 1500	4.5	2.5

8 OVERLAY DESIGN

8.1 The structural condition of the pavement can be assessed in different ways. One such method is the assessment of remaining life which is obtained by estimating the traffic

loads that the pavement was initially designed for and subtracting from them the traffic loads that have already been carried by the pavement. Some other methods estimate the remaining life of the pavement directly from the critical stress or strain levels in the present condition, without taking into account the volume of traffic already carried. Another approach is to compare the moduli of the present layers with those the layers were expected to have initially. There are also procedures which correlate the deflections or deflection bowl shape parameters with the remaining life of the pavement. In these guidelines, the method in which the remaining life of the pavement is estimated from the critical strains computed for the present condition of the pavement is adopted.

8.2 Any method of remaining life estimation will have its limitations and the results cannot automatically be accepted. It is, hence, very important that the estimations be compared with other indicators of the structural condition such as surface distress data, test pit inspection, coring data, etc., to check whether all these data give similar indications.

8.3 Performance Criteria

The layer moduli of in-service pavement backcalculated from FWD deflection data are used to analyse the pavement for critical strains which are indicators of pavement performance in terms of rutting and fatigue cracking. The following approach is proposed for design of bituminous overlays for existing flexible pavements. The mechanistic criteria (fatigue and rutting) adopted in the Indian Roads Congress guidelines (IRC:37-2012) for design of flexible pavements form the basis for the overlay design method. Performance models adopted in these guidelines are given below.

8.3.1 Fatigue in bituminous layer

Fatigue model for 90 percent reliability is given as Equation 16

$$N_f = 0.711 \times 10^{-04} \times [1/\epsilon_t]^{3.89} \times [1/M_R]^{0.854} \quad \dots (16)$$

where,

- N_f = fatigue life in standard axle load repetitions;
- ϵ_t = maximum tensile strain at the bottom of bituminous layer;
- M_R = Resilient modulus of bituminous mix, MPa

8.3.2 Rutting in subgrade

Rutting model for 90 percent reliability level is given by Equation 17

$$N = 1.41 \times 10^{-8} \times [1/\epsilon_v]^{4.5337} \quad \dots (17)$$

8.4 The following are the steps to be followed for design of overlays for Indian highways based on FWD evaluation.

- i) Measurement of surface deflections of homogeneous section of the in-service pavement using FWD

- ii) Normalization of the deflections to correspond to a standard load of 40 kN
- iii) Collection of information about layer type and layer thicknesses
- iv) Backcalculation of pavement layer moduli from the normalized deflections using an appropriate backcalculation software. Backcalculation will be done by considering the pavement to be a three layer system. All bituminous layers will be combined together. Similarly granular base and subbase layers may be combined. For pavements with modified subgrades, the modified layer may be treated as part of subgrade. Cemented subbases may be treated to be part of granular layer.

For pavements with cemented base, unless the pavement is being evaluated for its remaining life within a short period after construction, the cemented layer may be treated as part of granular layer

- v) Adjustment of the bituminous layer modulus (backcalculated) to a standard temperature of 35°C using the correction factors given by equations 4 and 5.
- vi) Adjustment of the subgrade and granular layer moduli to correspond to post-monsoon condition using equations 6 to 9
- vii) Selection of 15th percentile modulus (15% of the values will be less than this value) of each of the three layers considered for analysis
- viii) Analysis of the in-service pavement using linear elastic layer theory with the backcalculated (corrected) moduli and layer thicknesses collected from field as inputs. This includes computation of critical Strains (a) Horizontal Tensile Strain at the bottom fiber of bituminous layer and (b) Vertical Compressive Strain on top of subgrade. The loading configuration and the locations of critical strains considered for analysis will be similar to those adopted in IRC:37-2012.
- ix) Estimation of the remaining life of the pavement using the fatigue in bituminous layer and subgrade rutting performance criteria adopted in IRC:37-2012 given by equations 16 and 17. The strain values obtained in step viii will be used to estimate the remaining lives from fatigue and rutting consideration. Remaining life of the pavement will be the shortest of the lives obtained from bituminous layer fatigue, subgrade rutting and cemented base fatigue (in case of pavements with cemented base) criteria.
- x) For design of bituminous overlay, a trial thickness of overlay of an appropriate material has to be selected and the critical strains have to be evaluated. The modulus value of the bituminous overlay material may be selected as per the guidelines given in IRC:37-2012. Design overlay thickness can be selected by trial in such a way that the computed critical strains are less than the permissible limits given by the performance criteria for the

design traffic level considered. A typical design example is presented in **Appendix-IV** for better appreciation of the design approach. A format for compiling the backcalculated moduli is given as **Appendix-V**.

9 REFERENCES

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(Refer Clause 5.3.4)

Name of the Road :
 Section :
 No of lanes & Carriageway Type :

Subgrade Soil Type :

Note 1 : Identify the carriageway / lane position / paved shoulder
Note 2 : Classify as "Good", "Fair" or "Poor" based on the criteria given in Table 1 (minimum length of sub-section is 1.0 km except in case of localised failures requiring closer examination where the minimum length of the section should be 0.3 km)
Note 3 : Record any special or abnormal conditions such as flooding, submergence, failed section, previous failure history (if any), etc
Note 4 : It is advised that distress data may also be collected separately using suitable mapping formats as mentioned in Clause 5.3.4

Appendix-III

(Refer Clause 6.3.1)

KGPBACK SOFTWARE FOR BACKCALCULATION OF PAVEMENT LAYER MODULI

III.1 A number of softwares such as ELMOD, EVERCALC, BISDEF, NUS-BACK, MICH-BACK, MODULUS, PADAL, etc. are available for the backcalculation of pavement layer moduli from deflections measured using FWD. Some of these models use regression techniques for estimation of one or more moduli. Most of the traditional methods follow iterative approaches in which the moduli are varied in each step and the computed and measured deflections are compared. The iterations continue till satisfactory matching between the known and computed deflections is attained. The methods differ mostly in the techniques used for the successive selection of new set of modulus values. Other non-traditional techniques such as artificial neural networks and genetic algorithms have also been used for backcalculation.

III.2 KGPBACK, a specific version of BACKGA program, which was developed for the research scheme R-81 (2003) of the Ministry of Road Transport and Highways, is recommended in these guidelines for backcalculation. KGPBACK is a Genetic Algorithm based model for backcalculation of layer moduli. Because of the features of the search algorithm used in them, Genetic Algorithms (GA) have become popular as an optimization technique that has the ability to solve complex problems. The early use of GAs for backcalculation of pavement layer moduli was with the development of NUS-GABACK program by Fwa et al (1997). It was shown that NUS-GABACK performed comparably well against other programs and showed consistency in the accuracy of the solutions. The other GA based backcalculation method reported in literature was the program developed by Kameyama et al (1998) for the backcalculation of moduli of flexible and rigid pavements.

III.3 GA is a population based search and optimization technique. It maintains a population of individual solutions that compete amongst themselves based on the Darwin's survival of the fittest principle (Deb, 1995; Goldberg, 2000). Each individual solution (called chromosome) in the GA population is a finite length string code corresponding to a solution to the given problem. Each individual has a fitness value associated with it which is some measure of its closeness to the actual solution.

III.4 The population is initially generated randomly and the fitness of each individual in the population is calculated. Then the genetic operators are repeatedly applied to this population until the desired solution is found or specified number of iterations are over. A typical GA uses three operators: - reproduction or selection, crossover and mutation. In reproduction each individual in the population produces offsprings according to its fitness value. Thus, individuals with higher fitness contribute more offsprings in each generation. In crossover operation, two chromosome strings are randomly selected and their portions are exchanged with a probability P_c to form new individual. The crossover point about which the strings are exchanged is also selected randomly. In mutation operation, each bit in the string is flipped according to the probability P_m , which is usually very small.

III.5 Crossover and mutation operations on the mating pool generate offsprings. Fitness values are calculated for these offspring values. All these operations carried out sequentially

to complete one generation. The algorithm will run till the desired convergence is attained or till the maximum number of generations stipulated. The solution with the highest fitness value obtained in different generations is stored.

III.6 Backcalculation using GAs does not require seed moduli. Only the lower and upper domain bounds of the layer moduli (range of moduli) are required. Length of chromosome used to represent the variable is based on the required accuracy of backcalculated values. A chromosome length of '10' for each layer modulus is generally adequate.

III.7 The GA parameters such as population, maximum number of generations, probabilities of crossover and mutation are given as inputs to the program. Objective function OBJ is calculated from the calculated and measured deflection values. OBJ is the sum of squares of the relative error in deflections (difference between computed deflection and measured deflection divided by measured deflection) for all the sensor positions. Fitness of each solution set is evaluated using Equation III.1.

$$\text{Fitness} = 1 / (1 + \text{OBJ}) \quad \dots \text{ (III.1)}$$

The solution with the best fitness obtained in different generations is stored and given as the final output (backcalculated moduli) along with the corresponding fitness values.

III.8 Salient Features of KGPBACK

III.8.1 KGPBACK uses linear elastic layered theory for the analysis of pavements in its forward calculation algorithm. ELAYER computer program (Reddy, 1993), which uses linear elastic layered theory for the analysis of flexible pavements, was used as the forward calculation routine. It is recommended that the pavement be modeled as a three layer system. For this, layers with similar stiffness can be grouped together. Rough interfaces (with full bonding) are assumed between layers. KGPBACK, like most other backcalculation programs, does not backcalculate Poisson's ratio and thicknesses. Typical values of Poisson's ratio values are selected for the analysis as Poisson's ratio values (when chosen within practical range) are not expected to have any significant influence on the deflections. Thicknesses must be available from construction records, test pit data, cores or non destructive determination using techniques such as GPR.

III.8.2 Length of chromosome used to represent the variable is based on the required accuracy of backcalculated values. In KGPBACK, the length of chromosome was taken as the number of layers multiplied by 10, with each layer modulus being represented by a string of length "10".

III.8.3 It is important that GA parameters such as population size, maximum number of generations and probabilities of crossover and mutation, are selected in an optimal way so that the GA works satisfactorily for the given problem. Based on an extensive study conducted for the determination of optimal GA parameters for typical three layer pavement systems, the following GA parameters have been selected.

Population Size	:	60
Maximum Number of Generations	:	60
Crossover Probability	:	0.74
Mutation Probability	:	0.10

III.8.4 Ranges of different layer moduli are to be given as input to KGPBACK for backcalculation. These ranges are to be selected judiciously by experienced pavement engineering taking into consideration the approximate age of the pavement, visual assessment of the condition of bituminous layers, climatic conditions prevailing at the time of deflection measurements and any other information available from test pits, cores, DCP tests and laboratory tests conducted, if any. The following ranges of moduli are recommended for different layers for carrying out backcalculation using KGPBACK.

Subgrade Modulus

- If no information is available about subgrade 20 to 100 MPa
- If an estimate of in-situ CBR of subgrade can be obtained from DCP tests, classification of soil, laboratory CBR test, etc., the range of subgrade moduli can be taken as $5 \times \text{CBR}$ to $20 \times \text{CBR}$. Estimation of subgrade modulus range can also be made using the average of the surface deflections measured at radial distances of 1200 mm, 1500 mm and 1800 mm (if available) using the following expression.

$$E_{\text{subgrade}} \text{ (MPa)} = (1 - \mu^2) * P / (3.14 * r * w) \quad \dots \text{ (III.2)}$$

where,

P = total load (N) which can be calculated from applied load and radius of load contact area, r = average of radial distances (example 1200 mm, 1500 mm and 1800 mm), μ = Poisson's ratio of subgrade and w (mm) = average of surface deflections measured at 1200 mm, 1500 mm and 1800 mm (if available) radial distances.

The subgrade modulus estimated from the above methods can be used to narrow the range of moduli to be used in the backcalculation process which can improve the accuracy of the remaining backcalculated moduli. Experience of backcalculation of subgrade modulus from deflection bowls measured on Indian highways suggests that the backcalculated values are 20% more compared to the values estimated using Equation III.2. Hence, it is recommended that the lower bound value for subgrade modulus may be taken as $1.2 * (\text{modulus from Eq III.2}) * 0.8$ and the upper bound value can be $1.2 * (\text{modulus from Eq III.2}) * 1.2$.

Granular Layers (combined)	100 to 500 MPa
Bituminous Layer – thick layers without much cracking	750 MPa to 3000 MPa
Bituminous Layer in distressed condition (Fair to Poor)	400 MPa to 1500 MPa

Appendix-IV

(Refer Clause 8.4)

DESIGN EXAMPLE

Deflection measurements were made using FWD on a national highway in the month of January. Based on the deflection data and other parameters such as subgrade strength and pavement layer thicknesses different homogeneous sections have identified. This example presents the steps involved in the assessment of the in-service pavement and for design of bituminous overlay for a particular homogeneous section. The following data have been collected for the homogeneous section.

- Existing pavement has two bituminous layers with a total thickness of 170 mm. Total thickness of granular layers is 575 mm.
- Design Traffic = 100 msa
- Deflections measured at different locations of the homogeneous section are normalized for 40 kN standard load and are given the following Table.

Sl. No	Normalised Deflection at a Radial Distance (mm) of							Pavement Temperature (°C)
	0	300	600	900	1200	1500	1800	
1)	0.481	0.294	0.216	0.163	0.134	0.107	0.080	35
2)	0.478	0.317	0.231	0.186	0.156	0.130	0.106	35
3)	0.481	0.340	0.242	0.201	0.170	0.139	0.105	36
4)	0.500	0.321	0.233	0.198	0.151	0.130	0.093	36
5)	0.477	0.324	0.240	0.190	0.159	0.138	0.109	36
6)	0.485	0.319	0.230	0.194	0.152	0.141	0.101	37
7)	0.473	0.315	0.229	0.191	0.149	0.131	0.097	37
8)	0.460	0.301	0.223	0.188	0.151	0.130	0.093	38
9)	0.480	0.365	0.251	0.190	0.170	0.152	0.108	38
10)	0.487	0.327	0.245	0.187	0.161	0.148	0.102	38

- Layer moduli are backcalculated using KGPBACK program. The pavement has been modeled as a three-layer system with bituminous layer, granular layer and subgrade. Input needs to be given in the following sequence.
 - Single Wheel Load (N) and contact pressure (MPa)
40000 0.56
 - No. of deflection measuring sensors used in FWD
7 (for this example)
 - Radial distances (mm) where deflections were measured
0 300 600 900 1200 1500 1800
(geophone configuration used in the example)

- d) Measured Deflections (mm) starting from centre of load
0.481 0.294 0.216 0.163 0.134 0.107 0.080
- e) Layer thicknesses (mm) starting from top layer
170 575
- f) Poisson's ratio values of layers from top
 Suggested Poisson's ratio values are **0.5 0.4 0.4**
- g) Give practical ranges for each layer modulus (MPa)
 See guidelines for selecting moduli ranges. The moduli ranges used for the three layers in this example are :-
- Bituminous layer **750 to 3000 MPa**
 Granular Layer **100 to 500 MPa**
 Subgrade **As mentioned below**

Subgrade moduli estimated from Equation III.2 (of Appendix-III) for the ten locations are :- 59.6, 48.8, 46.2, 51.2, 47.2, 48.5, 50.9, 51.2, 44.4, 46.5 MPa. The corresponding ranges selected for subgrade modulus are :- 57.2 to 83.4; 46.9 to 68.3; 44.4 to 64.7; 49.1 to 71.6; 45.3 to 66.1; 46.6 to 67.9; 48.8 to 71.2; 49.1 to 71.6; 42.7 to 62.2; and 44.6 to 65.1 respectively.

- The moduli values backcalculated for all the ten test points of the homogeneous section are given in the following table

Sl. No	Layer Moduli (MPa)			Sl. No	Layer Moduli (MPa)		
	Bit	GB	Sub		Bit	GB	Sub
1)	1214.1	197.4	70.8	6)	991.9	250.1	57.2
2)	1022.7	254.8	57.3	7)	1040.3	252.4	60.6
3)	1458.2	214.6	55.0	8)	1313.0	245.5	60.1
4)	1295.5	195.4	60.6	9)	1669.4	200.5	53.7
5)	1240.5	245.1	55.4	10)	1247.1	229.4	56.1

- Bituminous layer moduli are corrected for a standard pavement temperature of 35°C using equations 4 and 5 and granular layers and subgrade moduli backcalculated from deflections collected in Winter are corrected for monsoon season using Equations 9 and 6 respectively. The corrected moduli are given in the following table.

Sl. No	Layer Moduli (MPa)			Sl. No	Layer Moduli (MPa)		
	Bit	GB	Sub		Bit	GB	Sub
1)	1214.1	171.7	59.7	6)	1085.2	217.1	46.3
2)	1022.7	221.0	46.4	7)	1138.2	219.0	49.7
3)	1524.7	186.9	44.1	8)	1504.4	213.3	49.2
4)	1354.5	169.9	49.7	9)	1912.8	174.5	42.7
5)	1297.0	213.0	44.5	10)	1428.9	199.7	45.2

- Selecting 15th Percentile moduli for the purpose of design, the design moduli of in-service layers are :- 1112, 173, 44.3 MPa respectively for bituminous, granular and subgrade layers
- The in-service three layer pavement system has been analysed with the above corrected moduli values. Standard dual wheel load of 20 kN on each wheel has been considered for analysis. Contact pressure of 0.56 MPa, spacing between dual wheels of 310 mm, Poisson's ratios of 0.5, 0.4 and 0.4 for the three layers starting from the top are the other inputs used. Tensile strain at the bottom of bituminous layer = 284.5 microstrains; Vertical strain on top of subgrade = 439.6 microstrains
- Remaining fatigue life of the pavement obtained using Equation 16 is 11.07 msa whereas the remaining rutting life as obtained from Equation 17 is 233.7 msa. The existing pavement is inadequate to carry the design 100 msa traffic from fatigue consideration

- **Selection of overlay thickness:**

The combination of existing pavement and overlay will be analysed as a four-layer system to ensure that fatigue and rutting criteria are satisfied for the assumed design traffic. Trial overlay thicknesses are selected and maximum tensile strain at the bottom of the existing bituminous layer has been computed using the thicknesses and moduli of various layers as inputs. Analysis of the pavement with a Bituminous Concrete overlay (with VG-30 binder) of 95 mm thickness yields a tensile strain of 159.8 microstrain and 208.3 vertical subgrade strain. Elastic modulus value of BC mix has been taken as 1695 MPa.

The following inputs are considered for calculation of critical strains, tensile strain at the bottom of total bituminous layer and vertical strain on top of subgrade

Layer thicknesses considered are :- 95, 170, 575 mm

Elastic Moduli used are :- 1695, 1112, 173, 44.3 MPa

Poisson's ratio values used in analysis are :- 0.5, 0.4, 0.4, 0.4

The fatigue life for this overlaid pavement will be 104.4 msa and rutting life will be 690.5 msa. Hence, design overlay thickness is 95 mm of Bituminous Concrete with VG-30 binder.

(Refer Clause 8.4)

Name of the Road

No of lanes & Carriageway Type

Homogeneous Section

From : To

Note 1 : Identify the carriageway / lane position / paved shoulder

Note 2 : Deflections may be normalised as suggested in Clause 4.4

Note 3 : Other sensor configurations mentioned in Clause 4.6 can also be adopted

Note 4 : Bituminous Layer Modulus may be corrected for Temperature as per Clause 6.4. Granular and subgrade layers may be corrected for season as per Clause 6.5